CALCULATIONS FOR:

Waterbury, VT IM 089-2 (43) Re-Advertised
Item 208.4, Cofferdams at Pier 1 and Pier 2

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Prepared by:

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Plans and Specifications:

Contract Plans from VTrans

Design References:

United States Steel Sheet Piling Design Manual, 1984
Caltrans Trenching and Shoring Manual, 2011
LRFD Steel Design, 2nd Ed., Segui, 1999
LRFD Manual of Steel Construction, 2nd Ed., AISC, 1998

Description of Cofferdams:

Cofferdams are to be installed for the construction of piers 1 and 2 on the new bridge 46A. Sheet piling will be driven to refusal at bedrock to encompass the new pier footing and existing pier column. Internal walers and bracing will be installed as excavation proceeds.

At pier 2, a tremie seal will be placed if dredging yields excessive seepage, uncontrollable by conventional means. A separate submittal will be provided if a seal is needed.

Cofferdam installation, excavation, pier construction, and backfill will occur with the existing bridge still in service. After removing cofferdam the bridge will be closed and piers demolished to 2' below grade.

Design Method:

Allowable stress design is used throughout. The allowable structural stresses will generally be limited to 0.6 * the yield or other critical stress. For soil pressures, the passive pressure, including a reduction factor appropriate to the value of δ/Φ, is divided by a factor of safety = 1.3. For LRFD portions of steel design checks, factor loads by 1.5 (with Φ typ = 0.9, this gives 0.9/1.5 = 0.6 allowable stress)

Soil Properties:

See borings B103 and B104 for Pier 1, and B105 and B106 for Pier 2. This is a loose-to-medium silt/sand soil to shallow rock, taken as cohesionless, with properties in the cofferdam zones estimated as follows:

Compactness	*Very Loose*	*Loose*	*Medium*	*Dense*	*Very Dense*
Relative density D_d	0	15%	35%	65%	85% 100%
Standard penetration resistance, N = no. of blows per foot	0	4	10	30	50
ϕ (degrees) *		28	30	36	41
Unit weight, pcf					
moist	<100	95-125	110-130	110-140	>130
submerged	< 60	55-65	60-70	65-85	> 75

*highly dependent on gradation

Table 2 - Granular soil (after Teng[1])

Pier 1:
N = 10 blows/ft (uncorrected, conservative)
Unit weight γ = 110 psf
Soil friction angle Φ = 30˚
Sheet friction angle δ = 15˚

For the ground angle β = 0˚ (equipment pad area and bottom of dredged cofferdam), using log-spiral chart:
$\beta/\Phi = 0.0$
$K_a = 0.32$
$\delta/\Phi = 0.5$, so from the reduction factor chart:
$K_p = 6.5 * 0.746 / 1.3 = 3.73$, including a factor of safety of 1.3

Pier 2:
N = 7 blows/ft (uncorrected, conservative)
Unit weight γ = 110 psf
Soil friction angle Φ = 29˚
Sheet friction angle δ = 15˚

Ground slope at the faces of cofferdam is not significant, as the slope is generally parallel with the wall.
$\beta/\Phi = 0.0$
$K_a = 0.33$
$\delta/\Phi = 0.5$, so from the reduction factor chart:
$K_p = 6.0 * 0.746 / 1.3 = 3.44$, including a factor of safety of 1.3

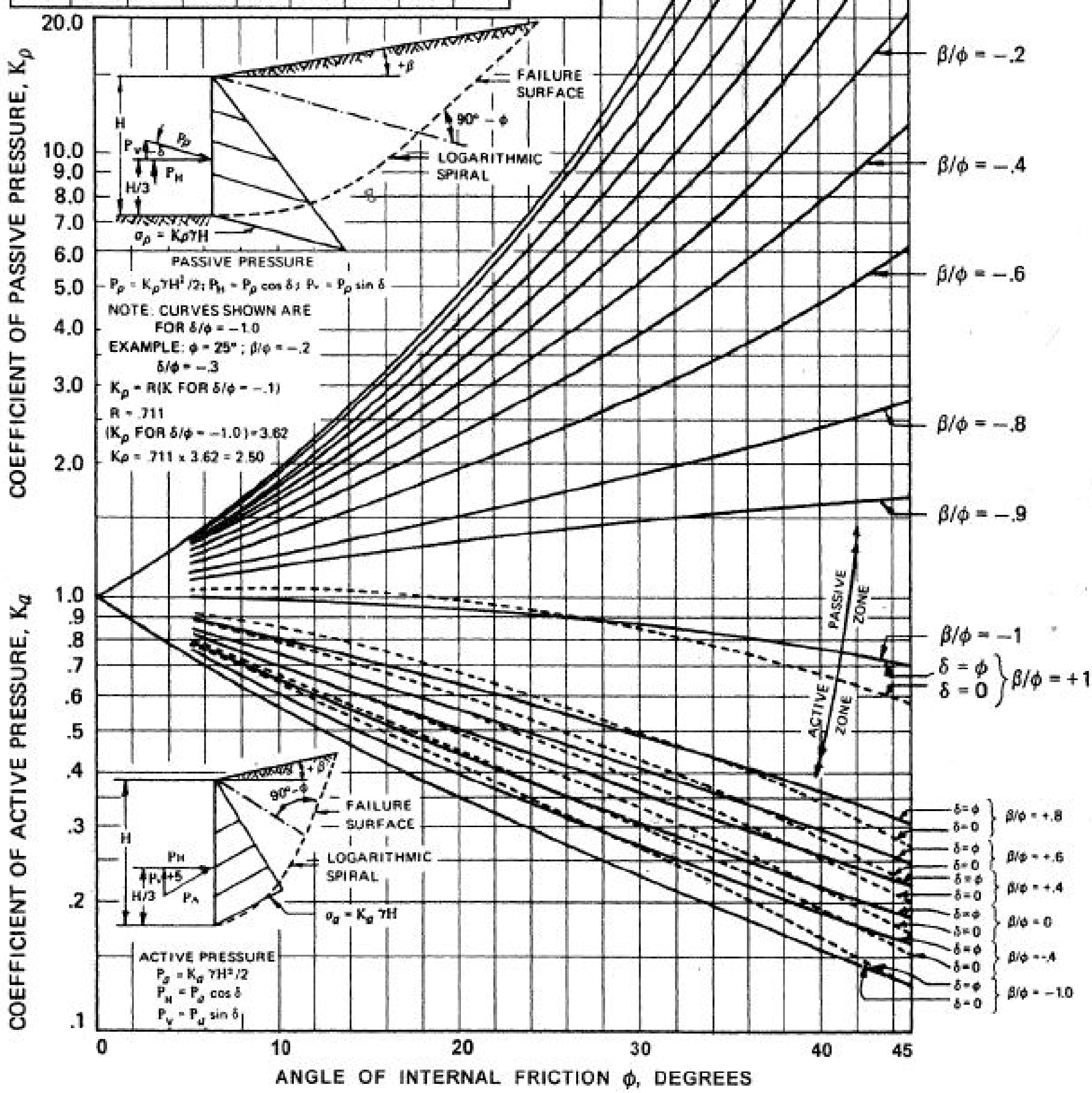
The pier 2 soil is slightly less favorable. Use these values for cofferdam design.

The water elevation is taken as el. 425', based on OHW in the plans, and river water levels in 2015 not exceeding el. 425'. Work will be done in late fall/winter, when water levels are low.

typ FoS on Kp is 1.2-1.5   eg: Kp = Kp · R · 1/1.2

REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$

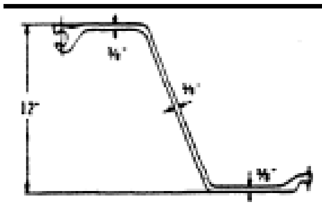
ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	-0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174



Steel Piling properties:

Steel used in the sheet piling and H-piles has a yield stress of 50 ksi; allowable bending stress = 50 ksi *0.6 = 30 ksi.

For sheet piling, use PZ27 or equal. S (per foot of wall) = 30.2 in^3, so allowable bending moment M_{allow} = 30 ksi * 30.2 in^3 = 906 k-in / 12 = **75.5 k-ft = M_{allow} on sheeting**.

Profile	Section Index		Distri Rolle	Driving Distance per Pile	Weight		Web Thickness	Section Modulus		Area	Moment of Inertia	
					Per Foot	Per Square Foot of Wall		Per Pile	Per Foot of Wall	Per Pile	Per Pile	Per Foot of Wall
				In.	Lbs.	Lbs.	In.	$In.^3$	$In.^3$	$In.^2$	$In.^4$	$In.^4$
	Interlock with and with PSA	PZ27	H.	18	40.5	27.0	3/8	45.3	30.2	11.91	276.3	184.2

Crane Load:

The largest crane available will be checked. This is a 138 HSL, 80-ton crawler crane (weight is described below). The pick load is taken as the heaviest of the following conditions:

ICE 416 vibratory hammer:
Hammer and hose: 10,175 lb.
60' long HP12x53 pile: 3,180 lb (governs over 40' sheet * 40.5 lb/ft)
Allowance for hook and rigging: 500 lb.
Total = 13,900 lb pick

Clamshell bucket:
4,000 lb bucket + 2 CY soil @ 120 pcf = 10,480 lb pick

Gross crane weight = 161,804 lb (see below output).

An excavator (CAT 429) has a gross weight of 70,000 lb. The crane load governs.

From the sketch below and ground bearing pressure program output, keep the pick radius at a maximum of 40', to provide even ground loading onto the crane mats. Chart capacity at 40' = 35,000 lb with max counterweight, ok.

Crane pressure is evenly distributed on crane mats, with a crane footprint of 20'x17'. Average ground bearing pressure = 162 k / (20' * 17') = 0.47 ksf, starting 2' from wall, extending 17' wide.

Link-Belt Constructon Equipment Co., Lexington, Kentucky - 138 HSL - Ext Gage   Unit: Englisl

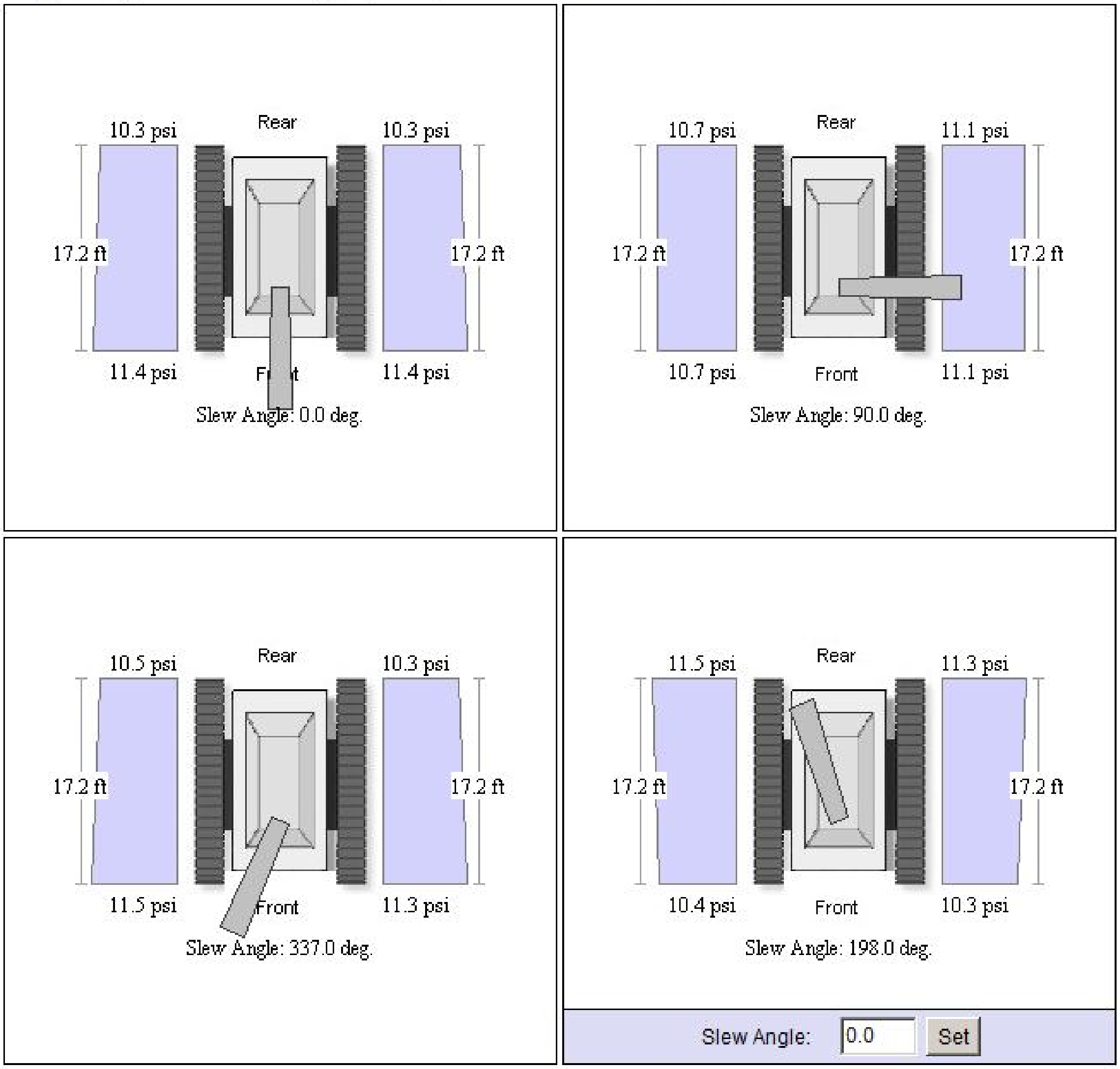
Model 138 HSL - Ext Gage Lattice Boom w/ ABC CTWT w/ 44" X 54" OT Boom

w/ 36" shoes

13900 lb load @ **40** ft radius, pick from **Boom**   **110** ft main boom

70.5° boom angle

161,804 lb gross vehicle weight (GVW)   Date: **8/6/2015** - v1.0



Slew Angle For Max Ground Bearing Pressure   Click & Drag the Boom or Input Slew Angle

The equipment surcharge horizontal loads can be modeled using a few line segments, based on the Boussinesq equation for lateral loading:

Calculation of lateral retaining wall pressures due to a strip load

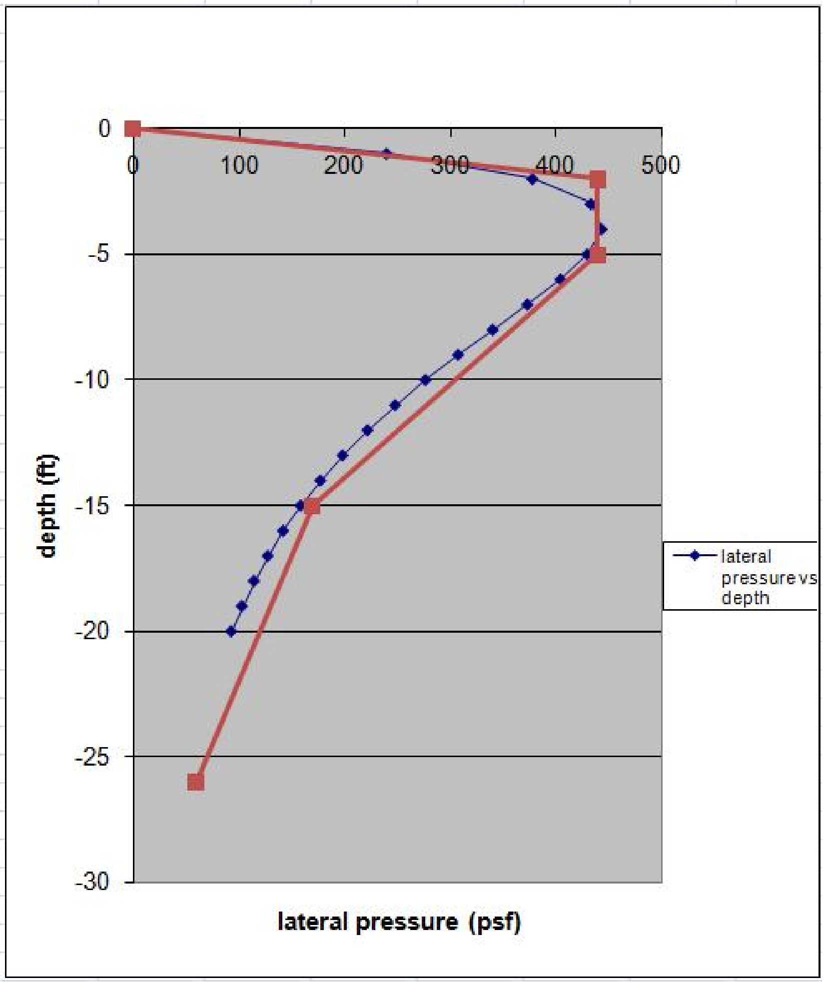
After Boussinesq Equation, after Teng

width of strip load (ft)	17
distance from wall edge to strip edge (ft)	2
q = psf of strip load	470

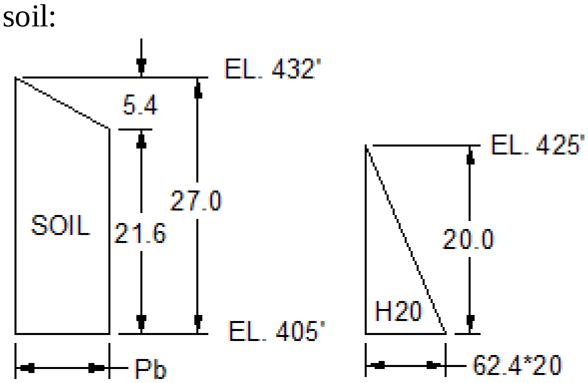
Load Model:

depth (ft)	$\sigma_{H\ (psf)}$
0	0
-2	440
-5	440
-15	170
-26	60

depth (ft)	β (rad)	α (rad)	$\sigma_{H\ (psf)}$
0	0	1.570796	0
-1	0.411065	1.475845	240.4062
-2	0.680521	1.382575	378.7003
-3	0.826192	1.292497	434.0212
-4	0.899652	1.206817	444.1109
-5	0.932966	1.126377	430.671
-6	0.943167	1.05165	405.1633
-7	0.939506	0.982794	374.0122
-8	0.927295	0.91972	340.9913
-9	0.909753	0.86217	308.3305
-10	0.888923	0.809784	277.301
-11	0.866147	0.762147	248.5644
-12	0.842331	0.71883	222.3974
-13	0.818097	0.679414	198.8403
-14	0.793873	0.643501	177.7949
-15	0.769955	0.610726	159.0882
-16	0.746548	0.580756	142.513
-17	0.723788	0.553294	127.8527
-18	0.701761	0.528074	114.8962
-19	0.680521	0.504861	103.4456
-20	0.660094	0.483447	93.32042



The internally braced cofferdam uses multiple rows of walers and is relatively rigid. Model the lateral soil loading using trapezoidal method, after Teng. For loose sandy soil:



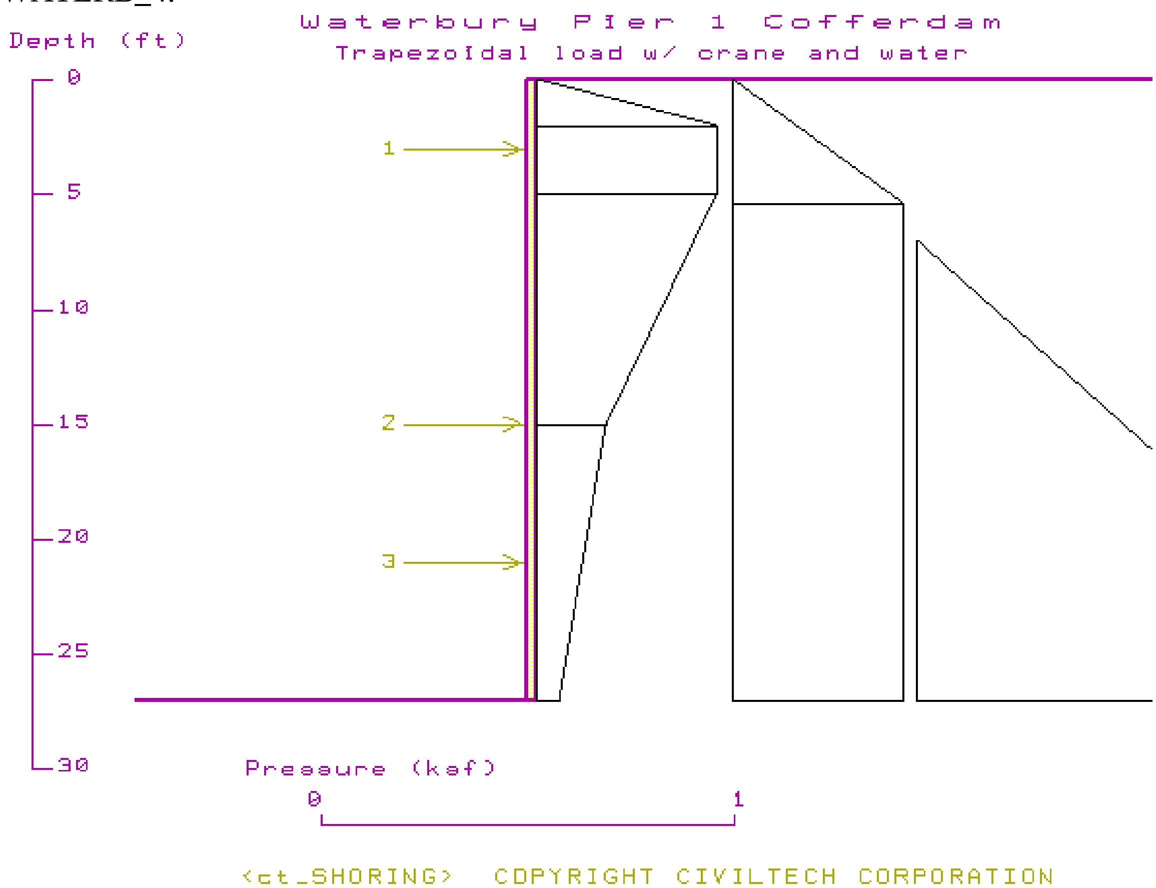
$P_b = 0.8*K_a*\gamma_e*H*\cos(\delta)$

$K_a = 0.33$,

γ_e = avg effective wt = [110 pcf*5.4' + (110 – 62.4) pcf*21.6'] / 27' = 60 pcf

P_b = 0.8 * 0.33 * 60 pcf * 27' * cos(15˚) = 413 psf

Distribute load to walers modeling sheeting as a beam, including equipment, soil, and water loading. Do not include toe resistance at bedrock. Shoring output for WATERB_4:



Waterbury Pier 1 Cofferdam
Trapezoidal load w/ crane and water

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******************************SUMMARY*********************************
EXCAVATION DEPTH =   27.00       INPUT FILE = WATERB_4.BRC
PILE LENGTH =   27.00            EMBEDMENT =    0.00
MAXIMUM MOMENT =    9.09         AT DEPTH =   21.00
BRACE REACTIONS OR TIEBACK HORIZONTAL COMPONENTS :
BRACE  1     REACTION =     6.36  AT DEPTH    3.50
BRACE  2     REACTION =     4.33  AT DEPTH   14.50
BRACE  3     REACTION =    18.04  AT DEPTH   21.00

******************************INPUT DATA*******************************
DRIVING PRESSURE (ACTIVE, WATER AND SURCHARGE PRESSURE):
```

Pres. No.	X Top	Pres. Top	X Bot.	Pres. Bot.	Width (Spacing)
1	0	0	2	.44	1
2	2	.44	5	.44	1
3	5	.44	15	.17	1
4	15	.17	27	.06	1
5	0	0	5.4	.413	1
6	5.4	.413	27	.413	1
7	7	0	27	1.25	1

X -- VERTICAL DISTANCE FROM GROUND SURFACE

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*********************************NOTE*******************************
FORCE -- kip; PRESSURE -- ksf; MOMENT -- kip-ft; LENGTH -- ft
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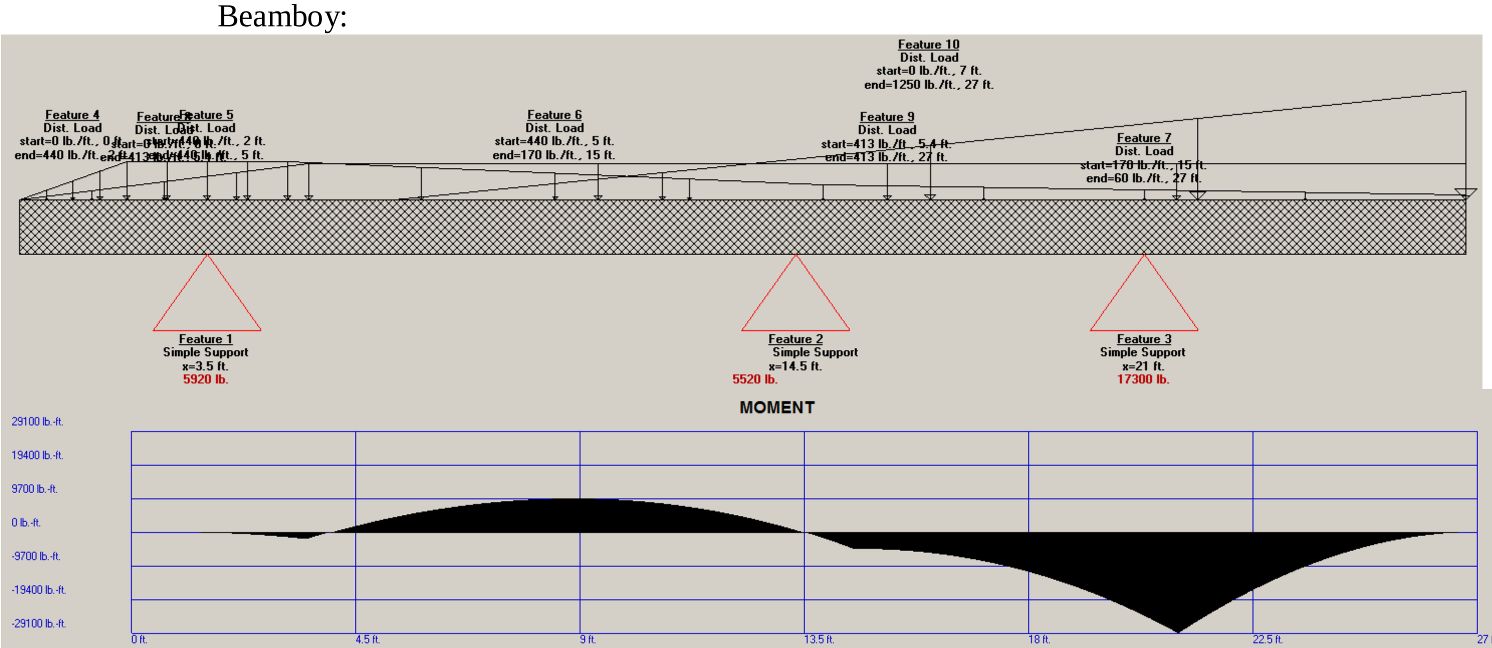
The above loading ignores any possible toe resistance, which would reduce waler loading.

Sheet pile bending shown above is inaccurate. Check bending & reactions with Beamboy:



Mmax = 29.1 k-ft << 75.5 k-ft. **Sheeting ok in bending**.
Reactions: R1 = 5.92 k, R2 = 5.52 k, R3 = 17.30 k, similar to above, reactions check.
Sum of reactions = 28.74, check.

Check cofferdam for removal of bottom waler line after pouring subfooting:

Remove bottom brace; take support to subfooting at 1' below top of subfooter, el. 409' = 23' depth. From WATERB_5:

```
Waterbury Pier 1 Cofferdam
Trapezoidal load w/ crane and water
****************************SUMMARY*********************************
EXCAVATION DEPTH =   27.00       INPUT FILE = WATERB_5.BRC
PILE LENGTH =   27.00            EMBEDMENT =    0.00
MAXIMUM MOMENT =   13.22         AT DEPTH =   23.00
BRACE REACTIONS OR TIEBACK HORIZONTAL COMPONENTS :
BRACE  1    REACTION =    6.36  AT DEPTH    3.50
BRACE  2    REACTION =    8.57  AT DEPTH   14.50
BRACE  3    REACTION =   13.79  AT DEPTH   23.00

****************************INPUT DATA******************************
DRIVING PRESSURE (ACTIVE, WATER AND SURCHARGE PRESSURE):
```

Pres. No.	X Top	Pres. Top	X Bot.	Pres. Bot.	Width (Spacing)
1	0	0	2	.44	1
2	2	.44	5	.44	1
3	5	.44	15	.17	1
4	15	.17	27	.06	1
5	0	0	5.4	.413	1
6	5.4	.413	27	.413	1
7	7	0	27	1.25	1

X -- VERTICAL DISTANCE FROM GROUND SURFACE

```
*******************************NOTE*********************************
FORCE -- kip; PRESSURE -- ksf; MOMENT -- kip-ft; LENGTH -- ft
```

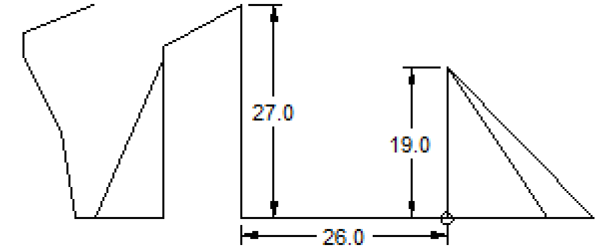
Waler 1 reaction = 6.36 k/ft, same as above
Waler 2 reaction = 8.57 k/ft, governs over 4.33 k/ft above
Waler 3 reaction = 13.79 k/ft taken up into subfooting.

Check minimum sheet pile bearing on subfooter for allowable concrete pressure:
13.79 k / 0.5 ksi allowable = 27.58 in2 over a 12" wide section of pile.
27.58 in2 / 12" = 2.3" height of sheet pile bearing on concrete << 18" + for toe of sheets at rock, ok.

Note: need to brace from river side waler to subfooting before removing bottom waler line. For pile embedment in soil, check waler reactions without bottom support. R1 = -4.3 k, so soil must take some load, approx 5 k per foot. For 3' of compacted fill with properties γ' = 120- 62.4 = 57.6 pcf, Kp = 10*0.674/1.3 = 5.18.
Total soil resistance = (57.6*5.18+62.4) *3'/2 * 3' = 1.62 k resistance < 5 k, NG. Provide braces at the bottom waler line to newly-placed subfooting in the same (3) locations as the waler braces.

At Pier 2, check unbalanced loading on cofferdam:

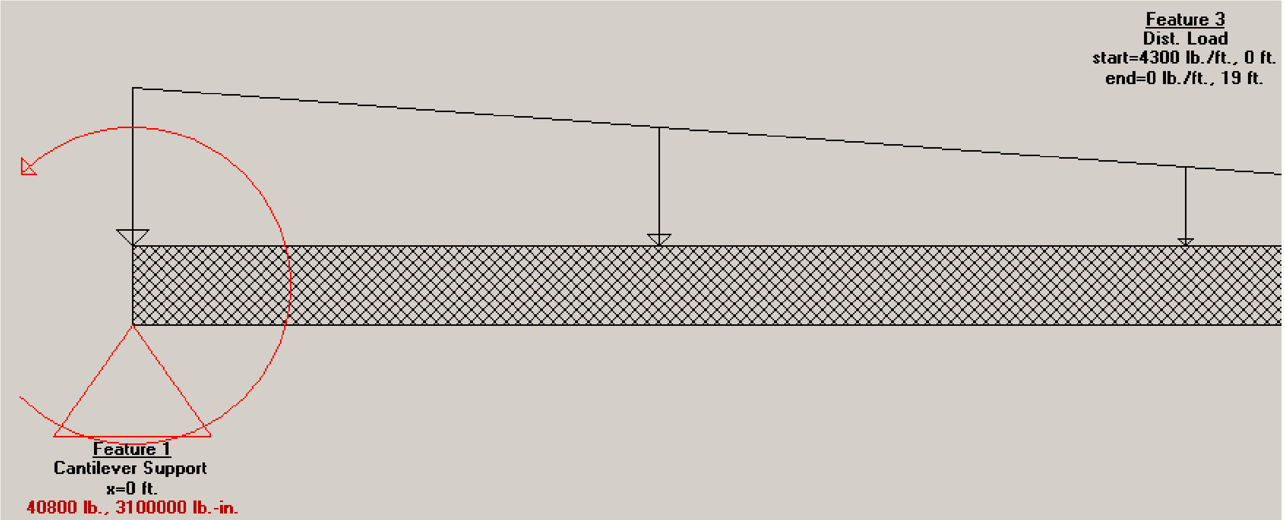
The grade at pier 2 drops off towards the river, with no equipment loading on the river side. Check passive resistance of cofferdam.



Horizontal Loading:

Horizontal active load: find by summing above waler reactions = 6.59 k + 4.02 k + 18.12 k = 28.73 k per foot of wall

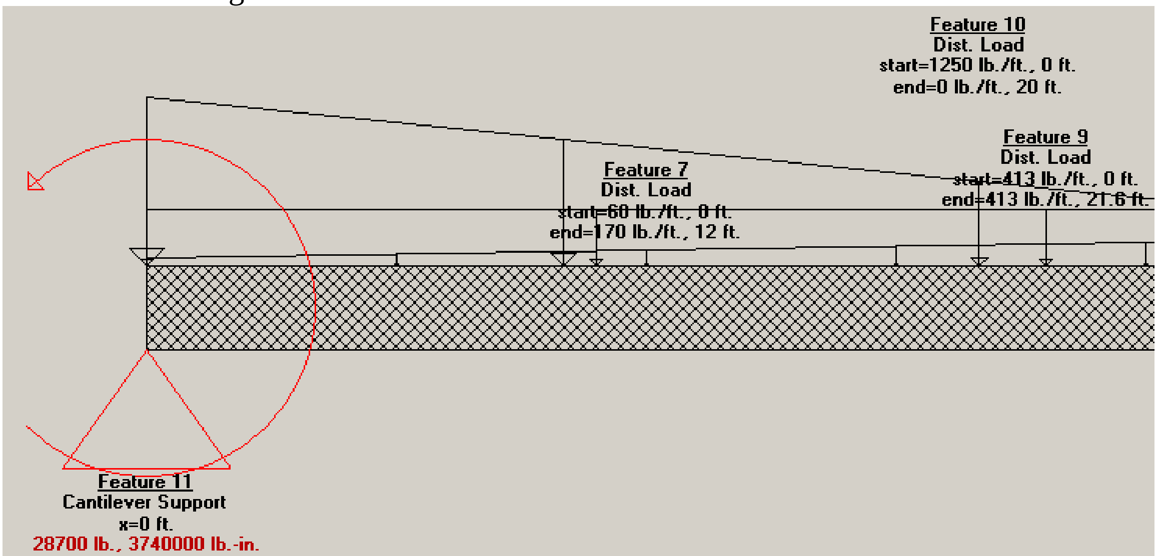
Horizontal passive load: take grade at el. 424’, typical where grade on high side is el. 432’. Take water at el 424’ as well, 1’ lower than on bank side.
Passive earth pressure = (110 – 62.4 pcf) * 3.44 + 62.4 = 226.1 psf/foot of depth.
226.1 * 19’ = 4,296 psf at 19’ depth



Resisting force = reaction =
40.8 k (including 1.3 FoS for passive pressure) > 28.73 k, ok in sliding.

Overturning about outer toe point:

Active overturning moment:



Active overturning moment for horizontal soil pressure = 3,740,000 lb-in / 12,000 = 312 k-ft

Active earth and equipment pressures have a vertical component due to wall friction.

Total horizontal component = 2*0.44/2 + 3*0.44 + 10*0.305 + 12*0.115 + 5.4*.413/2 + 21.6*0.413 = 16.22 k

Total downward vertical component = horizontal component * tan(δ) = 16.22 k * tan(15) = 4.35 k

Total reduction in overturning moment = 4.35 k * 26' = 113 k-ft
Total overturning moment = 312 k-ft – 113 k-ft = 199 k-ft

Passive resistance to overturning:
From above, the resisting moment = 3,100,000 lb-in / 12,000 = **258 k-ft (including 1.3 FoS) > 199 k-ft, ok in overturning.**

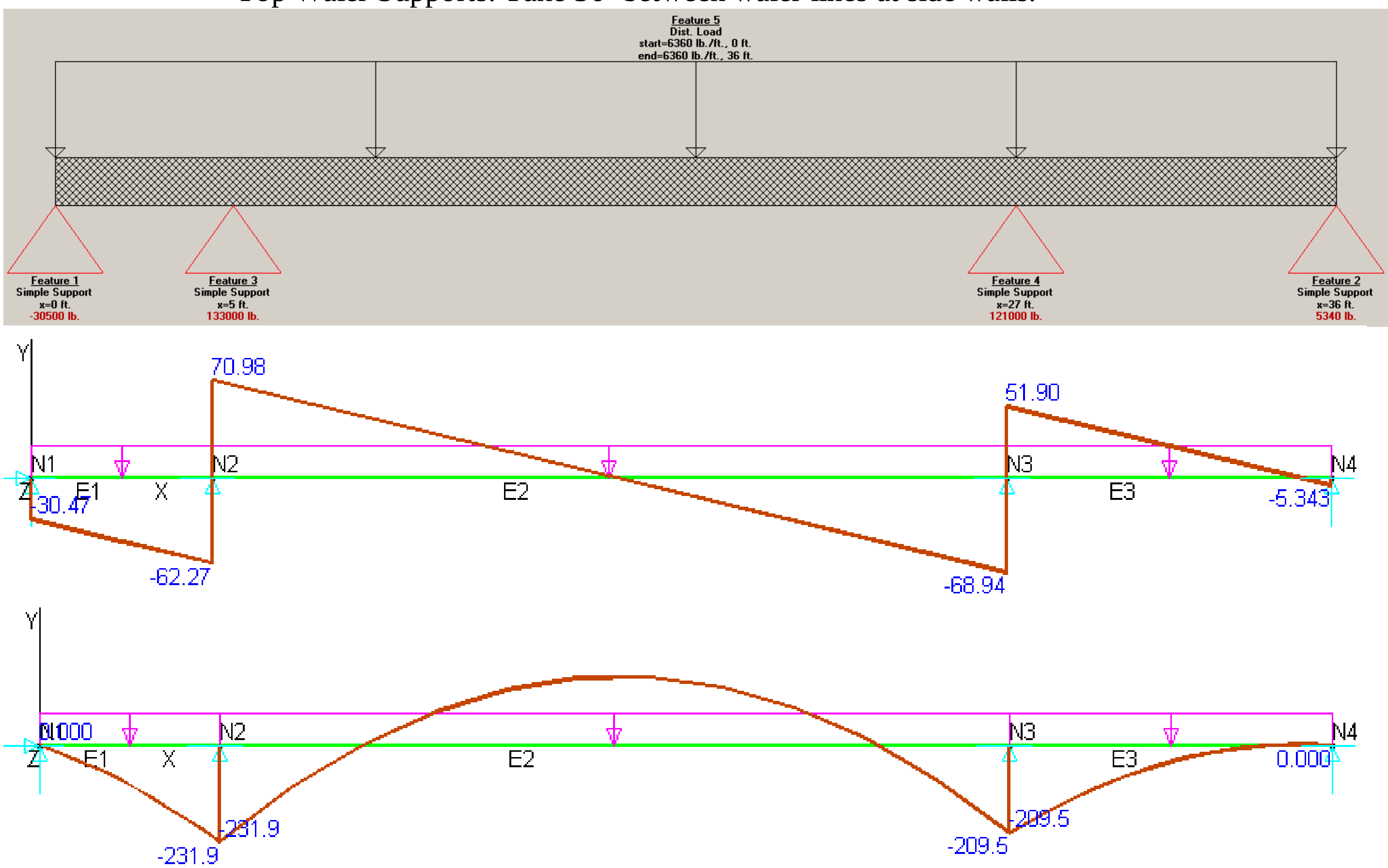
Check strength of sheeting due to load from top waler brace (see below calcs):
Say depth of fixity is at second waler line: Mmax = 6.59 k * (15' – 3.5')
= 76 k-ft = 910 k-in.
Sreq = 910 k-in/30 ksi = 30 in^3 per foot of wall. **PZ27 has S = 30.2 in^3 per foot of wall, ok for original waler location. Additionally, locate river side waler line at el. 423', so it bears against the soil. Waler will also transfer some load through the side walls into the ground via soil friction.**

Check waler loading:

Initial determination of waler supports show the following spacings. The long side walers are checked, with the short side walers having more frequent braces and therefore lower loading.

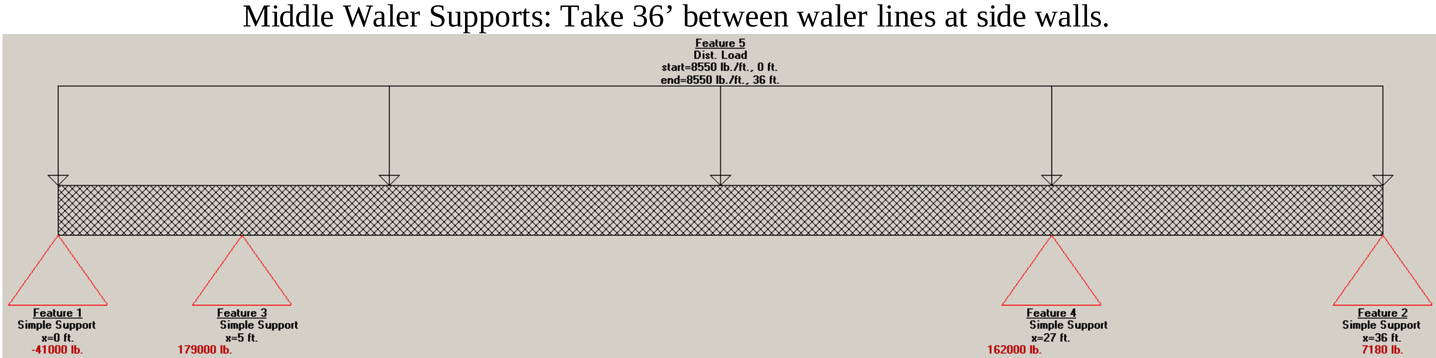
14’ max span for bottom waler.
22’ max span for mid and top waler.

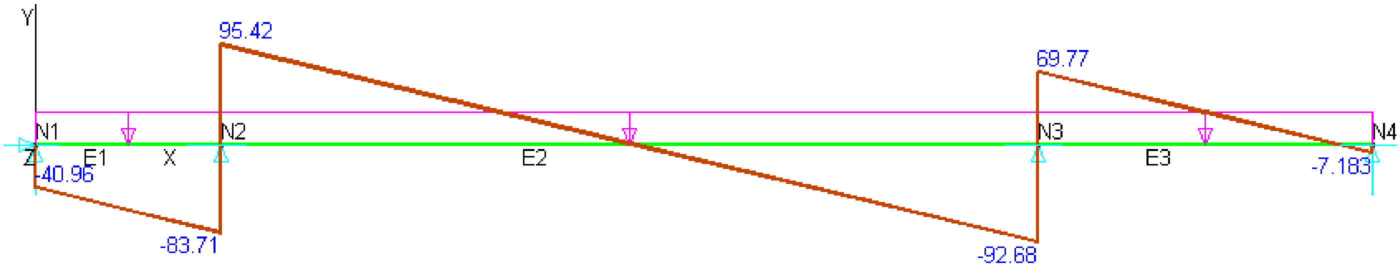
Top Waler Supports: Take 36’ between waler lines at side walls.

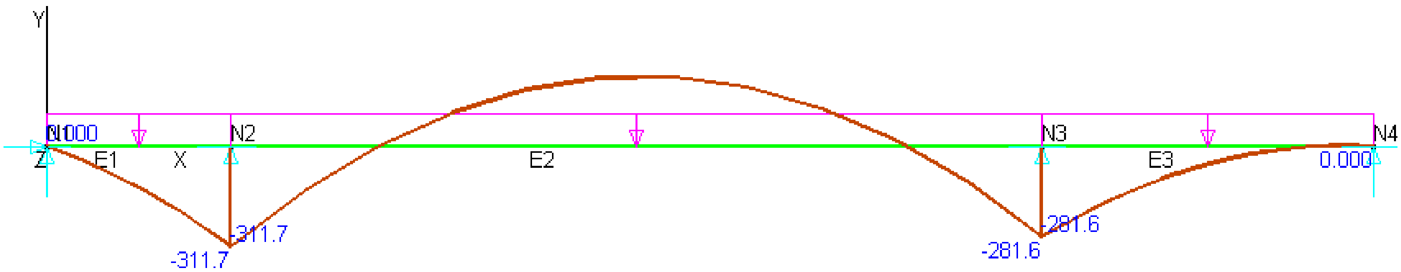


Max reaction = 133 k at knee brace
Vmax = 71 k.
Mmax = 232 k-ft = 2,784 k-in

Middle Waler Supports: Take 36’ between waler lines at side walls.

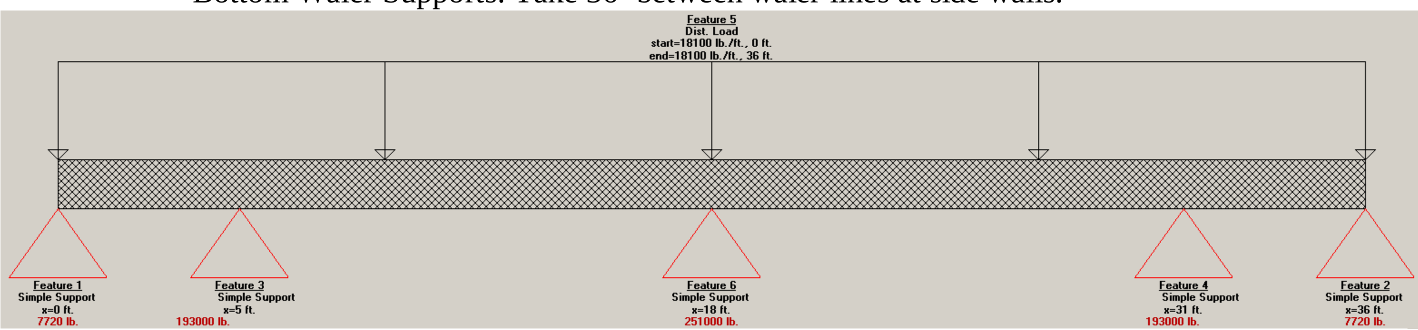


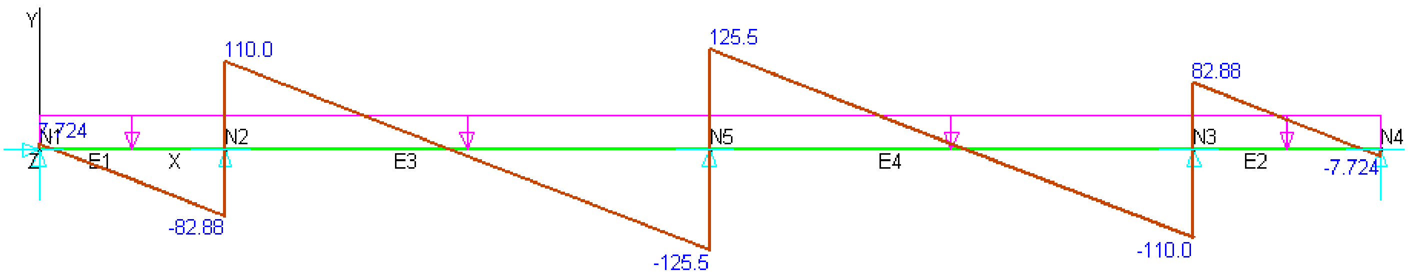


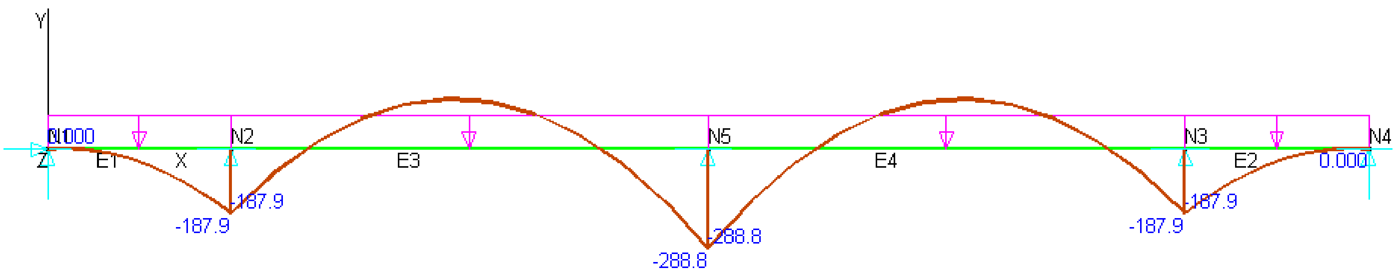


Max reaction = 179 k at knee brace
Vmax = 96 k.
Mmax = 312 k-ft = 3,744 k-in

Bottom Waler Supports: Take 36’ between waler lines at side walls.







Max reaction = 251 k at lower middle brace, 193 k at knee braces.
Vmax = 126 k
Mmax = 289 k-ft = 3,468 k-in

Check shear on middle and upper walers, using HP14x73 section:
HP sections do not have web instability, so V_{allow} = 0.6 (FoS) * 0.6 * F_y * A_w = 0.6 * 0.6 *50 ksi * 0.505"*11.25" = 102 k
Max shear above = 96 k. **Shear ok for a W14x73 or greater on middle and upper walers**.

Check shear on bottom walers, using HP14x102 section:
HP sections do not have web instability, so V_{allow} = 0.6 (FoS) * 0.6 * F_y * A_w = 0.6 * 0.6 *50 ksi * 0.705"*11.25" = 142 k
Max shear above = 126 k. **Shear ok for a W14x102 or greater on lower walers**.

Check bearing for walers:
Maximum R = 251 k, with the bearing length N = 14" for a 14" waler brace.
HP14x73 web yield at interior support: R_{allow} = 0.6 (FoS) *(5k + N) * F_y * t_w = 0.6*(5*1.1875" + 14") * 50 ksi * 0.505" = 302 k > 251 k, ok
Web cripple at interior support:
$R_{allow} = 0.6\,(FoS) *0.75*135*t_w^2*(1+3(N/d)*(t_w/t_f)^{1.5})*\sqrt{(F_y*t_f/t_w)}$ =
$0.6*0.75*135*0.505^2*(1+3(14''/13.61'')*(0.505''/0.505'')^{1.5})*\sqrt{(50\text{ ksi}*0.505''/0.505'')}$ = 447 k, ok. **Bearing on HP14x73+ waler is okay for lower and upper walers**.

Check waler bending:

Mmax for upper walers = 2,784 k-in. Sreq = 2,784 k-in / (50 ksi * 0.6 FoS) = 93 in3 if laterally braced. Waler is continuously braced at sheeting face, and braced at waler braces, where max negative bending occurs.

No LTB on walers, as they are connected to the sheeting.
Check HP14x73. Web is compact. Flange is non-compact, check FLB:
$\lambda = 14.4$, $\lambda p = 9.19$, $\lambda r = 22.29$. $\lambda p < \lambda < \lambda r$, so:
Mn = Mp – (Mp – Mr) * (λ – λp)/(λr – λp)
  Mp = Fy*Zx = 50 ksi * 118 in3 = 5,900 k-in
  Mr = (Fy-Fr) * Sx = 40 ksi * 107 in3 = 4,280 k-in
Mn = 5900 – (5900 – 4280) * (14.4 – 9.19) / (22.29 – 9.19)
Mn = 5,255 k-in; Mallow = 5,255 k-in*0.6,
For HP14x73, Mallow = 3,153 k-in (is < Mp) > 2,784 k-in,
HP14x73 waler ok in bending for upper waler.

Mmax for middle and lower walers = 3,744 k-in. Sreq = 3,744 k-in / (50 ksi * 0.6 FoS) = 125 in3 if laterally braced. Waler is continuously braced at sheeting face, and braced at waler braces, where max negative bending occurs.

No LTB on walers, as they are connected to the sheeting.
Check HP14x102. Web is compact. Flange is non-compact, check FLB:
$\lambda = 10.5$, $\lambda p = 9.19$, $\lambda r = 22.29$. $\lambda p < \lambda < \lambda r$, so:
$Mn = Mp – (Mp – Mr) * (\lambda – \lambda p)/(\lambda r – \lambda p)$
$Mp = Fy*Zx = 50$ ksi * 169 in3 = 8,450 k-in
$Mr = (Fy-Fr) * Sx = 40$ ksi * 150 in3 = 6,000 k-in
Mn = 8450 – (8450 – 6000) * (10.5 – 9.19) / (22.29 – 9.19)
Mn = 8,205 k-in; Mallow = 8,205 k-in*0.6,
For HP14x102, Mallow = 4,923 k-in (is < Mp) > 3,744 k-in,
HP14x102 waler ok in bending for middle and bottom walers.

As figured above, for HP14x102 walers, a 22' span for upper waler braces, and 14' for the bottom waler braces, is acceptable.

Check lower middle waler brace as a beam-column:

For the lower waler brace, try an HP14x117 beam-column. The transverse load = self-weight (taken as 117 lb/ft) + 50 lb/ft workers and incidental load = 167 lb/ft. For beam-column checks, use factored loads (1.50 * service loads). Axial = 251*1.50 = **377 k column load**, transverse = .167*1.50 = 0.251 k/ft. Transverse (vertical) loading is in the weak direction, with web laid horizontal.

Bearing load > 302 k web yield, so **include bearing stiffener at waler**.

The waler brace length is 25' between sheets – 2' for walers = 23' brace span. Assume pinned conditions at brace supports (worst case). Bending due to transverse loads: $PL^2/8 = 0.251$ k/ft * 23^2 ft^2 / 8 = 16.6 k-ft.

Beam bending is taken as the transverse load moment, plus the moment induced by a 5" eccentricity of the axial load. Mmax = 16.6 k-ft + 377 k*5/12 ft = 174 k-ft * 12 = **2088 k-in beam bending moment**

Strength as column:
Column design chart, HP14x117, 50 ksi, KL = 23':

ϕ:	0.85	
Pn:	945	k
ϕ*Pn =	803.25	k

Strength as a beam:

Mu =	2088	k-in	(imposed moment from calculations above)

Flange properties:

λp:	9.19	λ:	9.245342	λr:	22.29

Flange is non-compact, check strength by FLB (w/ bending in the weak direction)

Fy:	50	ksi	
Zy:	91.4	in3	
Mp =	4570	k-in	Fy*Zy
Fy-Fr:	40	ksi	
Sy:	59.5	in3	
Mr =	2380	k-in	(Fy-Fr)*Sy
Mn:	4560.748	k-in	Mp-(Mp-Mr) * (λ - λp) / (λr - λp)
ϕ:	0.9		
ϕ*Mn =	**4104.7**	**k-in**	< Mp, ok

Determine the state of LTB:

Lb =	276	in	
ry:	3.59	in	
Lp =	152.3	in	300*ry/sqrt(Fy)
X1:	3870	ksi	
X2:	0.000659	1/ksi^2	
Lr =	541.8	in	ry*X1/(Fy-Fr) * sqrt(1+ sqrt(1+X2(Fy-Fr)^2))

Lp < Lb < Lr, so check for inelastic LTB:

Cb:	1.14		for continuous load over simple supports
Mn =	4416.975		Cb*(Mp-(Mp-Mr)*(Lb-Lp)/(Lr-Lp))
ϕ*Mn =	**3975.278**	**k-in**	< Mp, ok

Find the Moment Amplification Factor for the column:

Pu:	377	k	(imposed axial load from calculations above)
Pe:	1664.498	k	π^2*E*I/L^2
MAF =	1.292816		1/(1-Pu/Pe)

Check Interaction Equation:
Pu/ϕPn = 0.469343 > 0.2, so check the following:
Pu / ϕPn + 8/9*(Mu* MAF /Mn) < 1.0 (for the lowest Mn found)
1.012581 = 1.0, beam-column is ok.

Check lower and upper knee braces as a beam-column:

For all knee braces, try an HP14x73 beam-column. For beam-column checks, use factored loads (1.50 * service loads). Knee braces are at 45˚ to wall. Max perpendicular load = 193 k * 1.5 FoS = 290 k

Axial load = 290 k /sin(45) = **410 k column load**,

The transverse load = 73 lb/ft self-weight + 50 lb/ft workers etc = 123 lb/ft. transverse = .123*1.50 = 0.185 k/ft. Transverse (vertical) loading is in the weak direction, with web laid horizontal.

The waler knee brace length is 11.5' = 12' max span. Assume pinned conditions at brace supports (worst case). Bending due to transverse loads: $PL^2/8$ = 0.185 k/ft * 12^2 ft^2 / 8 = 3.3 k-ft.

Beam bending is taken as the transverse load moment, plus the moment induced by a 5" eccentricity of the axial load. Mmax = 3.3 k-ft + 410 k*5/12 ft = 174 k-ft * 12 = **2090 k-in beam bending moment**

Strength as column:							
Column design chart, HP14x102, 50 ksi, KL = 12':							
ϕ:	0.85						
Pn:	1130	k					
ϕ*Pn =	960.5	k					
Strength as a beam:							
Mu =	2090	k-in	(imposed moment from calculations above)				
Flange properties:							
λp:	9.19	λ:	10.5	λr:	22.29		
Flange is non-compact, check strength by FLB (w/ bending in the weak direction)							
Fy:	50	ksi					
Zy:	78.8	in3					
Mp =	3940	k-in	Fy*Zy				
Fy-Fr:	40	ksi					
Sy:	51.4	in3					
Mr =	2056	k-in	(Fy-Fr)*Sy				
Mn:	3751.6	k-in	Mp-(Mp-Mr) * (λ - λp) / (λr - λp)				
ϕ:	0.9						
ϕ*Mn =	**3376.4**	**k-in**	< Mp, ok				
Determine the state of LTB:							
Lb =	144	in					
ry:	3.56	in					
Lp =	151.0	in	300*ry/sqrt(Fy)				
X1:	3400	ksi					
X2:	0.00109	1/ksi^2					
Lr =	493.2	in	ry*X1/(Fy-Fr) * sqrt(1+ sqrt(1+X2(Fy-Fr)^2))				
Lb < Lp, so no LTB; Mn = Mp							
Mn =	3940		Cb*(Mp-(Mp-Mr)*(Lb-Lp)/(Lr-Lp))				
ϕ*Mn =	**3546**	**k-in**	< Mp, ok				
Find the Moment Amplification Factor for the column:							
Pu:	410	k	(imposed axial load from calculations above)				
Pe:	5245.131	k	π^2*E*I/L^2				
MAF =	1.084796		1/(1-Pu/Pe)				
Check Interaction Equation:							
Pu/ϕPn =	0.426861	> 0.2, so check the following:					
Pu / ϕPn + 8/9*(Mu* MAF /Mn) < 1.0 (for the lowest Mn found)							
0.964048	**< 1.0, beam-column is ok.**						

Check waler connections:

The walers connect at right angles in the corners of the cofferdam. The max load on the corners from the bank side sheeting is 41 k. Loading on the short sides of the cofferdam is smaller; assume the same load as on the main face.
Resultant load at corners = 41 k /sin(45) = 58 k.

For 7/8" A325 bridge bolts, threads included; design as a bearing-type connection. Allowable load is taken as 0.6* LRFD ultimate loading.
R_{allow} per bolt = 0.6 FoS * 0.75 * 90 ksi * 0.601 in^2 = 24 k per bolt.
58 k / 24 k/bolt = 2.4 = 3 bolts required. (4 are being used in each location as a standard connection.)
Bolt spacing is about 6" minimum, and edge distance is about 3". Both are >3 diameters.
Hole bearing: R_{allow} per bolt = 0.6 * 0.75*2.4*0.875"*0.5"*58 ksi ultimate
R_{allow} = 27.4 k per bolt > bolt strength, ok.
Use **4 ea. 7/8" A325 bolts per connection at the waler corner connections**.

For the corner braces, the angle with the waler brace is 45 deg. The shear force is thus equal to the normal force, neglecting friction.

Upper brace: 133 k / 24 k/bolt = 6 bolts required.
Middle brace: 179 k / 24 k/bolt = 8 bolts required.
Bottom brace: 193 k / 24 k/bolt = 8 bolts required.

For the transverse brace at the bottom waler, there is little shear. 167 lb/ft * 12.5' = 2.1 k, negligible. Use 4 bolts in the connection.

Check waler brace bearing plates:
½" plate used in bolt checks above. Determine required base plate thickness from LRFD, section 11.
Use factored loads above, max load = 193 k.
t = L * √(2*Pu / 0.9*B*N*Fy)
L = max {m, n, λn'}
m = (N-0.95*d) / 2 = (21"-0.95*21") / 2 = 0.525"
n = (B-0.8*bf) / 2 = (14"-0.8*14") / 2 = 1.4"
λn' (λ = 1.0) = 1.0*√(d*bf)/4 = 1.0*√(21"*14")/4 = 4.29"
t = 4.29"* √(2*193k / (0.9*21"*14"*50 ksi)) = 0.73";
use ¾" plate for angle brace base plates, 21"x14" to match HP14 braces on a 45-deg angle.

For the bottom waler cross-brace:

$t = L * \sqrt{2*Pu / 0.9*B*N*Fy}$

$L = \max \{m, n, \lambda n'\}$

$m = (N-0.95*d) / 2 = (14''-0.95*14'') / 2 = 0.35''$

$n = (B-0.8*bf) / 2 = (14''-0.8*14'') / 2 = 1.4''$

$\lambda n' (\lambda = 1.0) = 1.0*\sqrt{d*bf}/4 = 1.0*\sqrt{14''*14''}/4 = 3.5''$

$t = 3.5''* \sqrt{2*377 \text{ k} / (0.9*14''*14''*50 \text{ ksi})} = 1.02''$;

use 1" plate for bottom cross-brace base plate, approx. 14"x14" to match HP14x117.

Welds at bearing plates:

max shear = 193 k. for ¼" weld,

allowable weld stress for A70XX = 70 ksi * 0.9 * 0.6 for shear * 0.6 FoS = 22.6 ksi.

193 k / 22.6 ksi = 8.54 in2 required weld area.

¼" weld has throat of 0.25"*sin45 = 0.176".

Require 8.54 in2 / 0.176" = 48", or 48/14" = 3.4 = 4 14" runs of weld.

¼" weld 1 side of each flange, and both sides of web.